# FATIGUE AND TENSILE CHARACTERISTICS OF BITUMINOUS PAVEMENTS AT LOW TEMPERATURES

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## SYNOPSIS

Pavement must perform through a wide range of temperatures. Most test methods and mix design procedures are based on elevated temperatures. Due to the drastic change in physical character of asphalt as temperature decreases, the critical service conditions may be at low temperatures.

This research applied a split cylinder tension test to typical bituminous pavement mixes at various low temperatures. As further verification of the importance of low temperature character, a series of flexure fatigue tests were carried out. The temperature range of zero to 40°F was investigated. The effect of asphalt content and grade is included. Due to the non-homogeneous nature of bituminous-aggregate mixtures, the results of the split cylinder test are in terms of work to cause failure rather than unit tension stress.

## INTRODUCTION

Tensile and fatigue properties of asphaltic concrete have not received much consideration in the design of asphalt pavements. During the recent years longitudinal cracking has developed in many high-type flexible pavements which cannot be accounted for by base or subgrade failures. Even though it is well known that the properties of asphalt are highly temperature susceptible, no mix or pavement design method takes this into account. Presently much research is being conducted on the flow properties with temperature and age. The usual temperature range of even these tests has been limited to the range from 140°F to 40°F. Practically no information about the properties of asphalt is available at temperatures below 40°F. The properties of bituminous mixes at low temperatures has not been investigated outside of research by Rader (1) in 1936, and in 1963 by Tons & Krokosky (2).

First, there is a lack of appropriate testing methods and second, the relationship between low temperature and flexible pavement failures has not been considered. In this research two testing methods are developed. The results of the testing indicate that low temperature properties are extremely important and should be accounted for in mix and pavement design procedures.

PRESENT PAVEMENT DESIGN CONSIDERATIONS

The design of flexible pavements is presently based on theoretical principles. However, equations accounting for all of the complex variables have not been developed for routine use. Consequently, most of the design procedures are of an empirical nature and are based upon arbitraty tests such as the California Bearing Ratio Method (3) or soil classification methods (4). Correlation values for performance as related to physical properties measured on field installations are used. In all of these methods the purpose of the upper layers of the pavement (subbase, base, surface) is considered to be the provision of a sufficient thickness of higher quality material so that the subgrade will be able to withstand applied wheel loads without undergoing distress. The vertical stress that exists in the subgrade has been the subject of many research reports.

These stresses have been found to be a function of the magnitude of the load, tire area, and the thickness of the pavement above the subgrade. For example, if a certain subgrade material can withstand an ultimate stress $\sigma_u$ , then knowing the tire area and wheel load it is possible to determine the thickness of higher quality material that has to be placed over the subgrade so that the vertical stress will be less than $\sigma_u$ .

The Kansas Method of design is based on theoretical stresses as determined by Boussinesq (5). Palmer and Barber (6) were the first to propose this type of pavement design method. It was assumed that the pavement was incompressible, in that all strains were elastic. That is, it was assumed that the base and wearing course followed the deformations of the subgrade without any change in the thickness of the pavement under load. The modulus of elasticity of the subgrade, the magnitude of the load, radius of contact of the tire are all interrelated and an allowable deflection criteria is used. No consideration is given to the variability of the base and wearing course properties. After the development of Burmister Theory (7) the method was revised to take into account the fact that a subgrade has a reduced strength when in a saturated condition. Also a modification was introduced to provide for the ratio of the modulus of the pavement system to that of the subgrade. Even then, the modulus of the pavement and the subgrade were assumed to be constant and independent of time and temperature.

The California Design Method that was developed by Hveem and Carmany (8) is based upon load repetitions, tire pressure, tire contact area and the cohesive properties of the bituminous surface. A resistance value is introduced to account for the fact that the pavement surface is in a confined state under load. Although load repetition is taken into account, no consideration is given to temperature effects.

Most states have methods that are modifications of these methods. Missouri and Alabama pavement design procedures conclude only the soil classification of the subgrade. As an example, a highway that has three hundred trucks per day over an A-7 subgrade must have 16 inches of pavement, whereas the same loading over an A-4 soil requires only 6 inches. Even in these methods the basic philosophy of flexible pavement design is present, that is, do not over-stress the subgrade.

Within the New England states, experience has been the main criterion for pavement thickness. In most cases a standard pavement section is used for all conditions. The state of Maine supplements experience with a frost criterion in areas of extreme cold.

Rigid pavement design methods do not depend as strongly on the properties of the subgrade, although they are important. In rigid pavement design, the slab is designed as a structural element and resists loads primarily by bending action. Westergard's Theory is utilized as the basis for determining the thickness requirements of rigid pavements. It is assumed that the pavement is a stiff elastic slab resting on an elastic subgrade. Stresses are computed for various loading conditions and the concrete slab designed to resist them. Basically rigid pavement design differs from flexible pavement design in that the modulus of the concrete slab is much greater than that of the subgrade material and the major portion of the load carrying capacity is derived from the slab itself. In flexible pavements the modulus of the surface course is only slightly greater than that of the base, which in turn, is slightly greater than that of the subgrade. Rigid pavement methods do take into account the fact that concrete is subject to fatigue. The allowable working stress is less than 50 percent of the modulus of rupture as determined by a flexure test.

In flexible pavements the surface course also constitutes a major structural element of the pavement. Results from the Western Association of State Highway Officials Road Test (WASHO) ( 9 ) and the American Association of State Highway Officials Road Test (AASHO) ( 10 ) indicates that one inch of asphaltic concrete surface is equivalent to two inches of crushed stone base. It was found that the thickness of the surface course did reduce the subgrade stress. It can be stated, with some reservation, that a portion of the load that is applied to the flexible layered system is resisted by beam action. As temperatures decrease, there may be a tendency for this beam action to increase due to increased rigidity of the surface course. There is a need for tests to evaluate the low temperature properties of pavements.

## THE SPLIT-CYLINDER TEST

Present mix design methods for the surface course consider stability density and permeability as criteria. The three most common types of mix design methods are based on the Marshall Test, the triaxial test and the Hveem Stabilometer. All three are conducted at 140$^{\circ}$F.

The mix design method presently used in Connecticut is based on the Marshall Stability Test which is an attempt to determine the optimum percentage of asphalt which, with a given gradation of coarse and fine aggregates, will have the maximum stability compatible with the desired air voids. Stability as used in the Marshall method of test is an empirical measure of mix resistance to deformation. As the aggregate particles are many orders of magnitude more rigid than the asphalt, the deformation must occur by rearrangement of the aggregate particles. Any rearrangement must be accompanied by cohesive or tensile stresses in the asphalt binder. At elevated temperatures such as 140°F, that is used in this test, the viscosity of the asphalt is low enough to permit relaxation of the tensile stresses by accompanying flow. In contrast at low temperatures, the asphalt within the mix has low ductility and stresses will build up until the tensile strength is reached and fracture results. Pavement failures in recent years have indicated that the surface course has been failing in tension. Passing wheel loads cause tensile stresses in the lower portion of the pavement under the tire contact area, and in the upper portion to either side of the tire. At low temperatures, the high viscosity of the asphalt prevents relief of the stresses by relaxation. Deflection measurements of surfaces on deep stone bases have been found to be nearly constant summer and winter. Forcing a cold pavement to assume the same curvature as warm pavement could cause substantially higher tensile stresses in the cold asphalt due to its decreased flow properties at low temperatures.

Additional tensile stresses are caused by a drop in pavement temperature. Asphalt and stone shrink much like other materials when cooled. However, the sharply differing coefficients of contraction create internal stresses within the pavement mix. The greater reduction in asphalt volume results in asphalt tension analogous to water tension caused by evaporation of water from soil.

Attempts to predict the interrelationship of temperature, strength and mix viscosity are defeated by the heterogeneous nature of an asphalt mix.

In order to determine such interrelationships, the low temperature tensile strengths were measured for various asphalt mixes.

The split cylinder test was used to determine low temperature tensile properties because of its applicability to testing brittle materials (such as concrete), the rapid rate of conducting the test, and the ease of assembling the apparatus.

The split cylinder test for concrete was developed independently by Carneiro and Barcellus (11) in Brazil and by Akazawa (12) in Japan. It has been used extensively in determining the tensile strength of concrete as shown in papers by Rusch and Vigerlust (13) and McNeeley and Lash (14).

Figure 1 and 2 shows the test specimen with the applied diametrical load. The equations for the state of stress within the cylinder are found in texts by Den Hartog (15), Timoshenko (16), and Frocht (17).

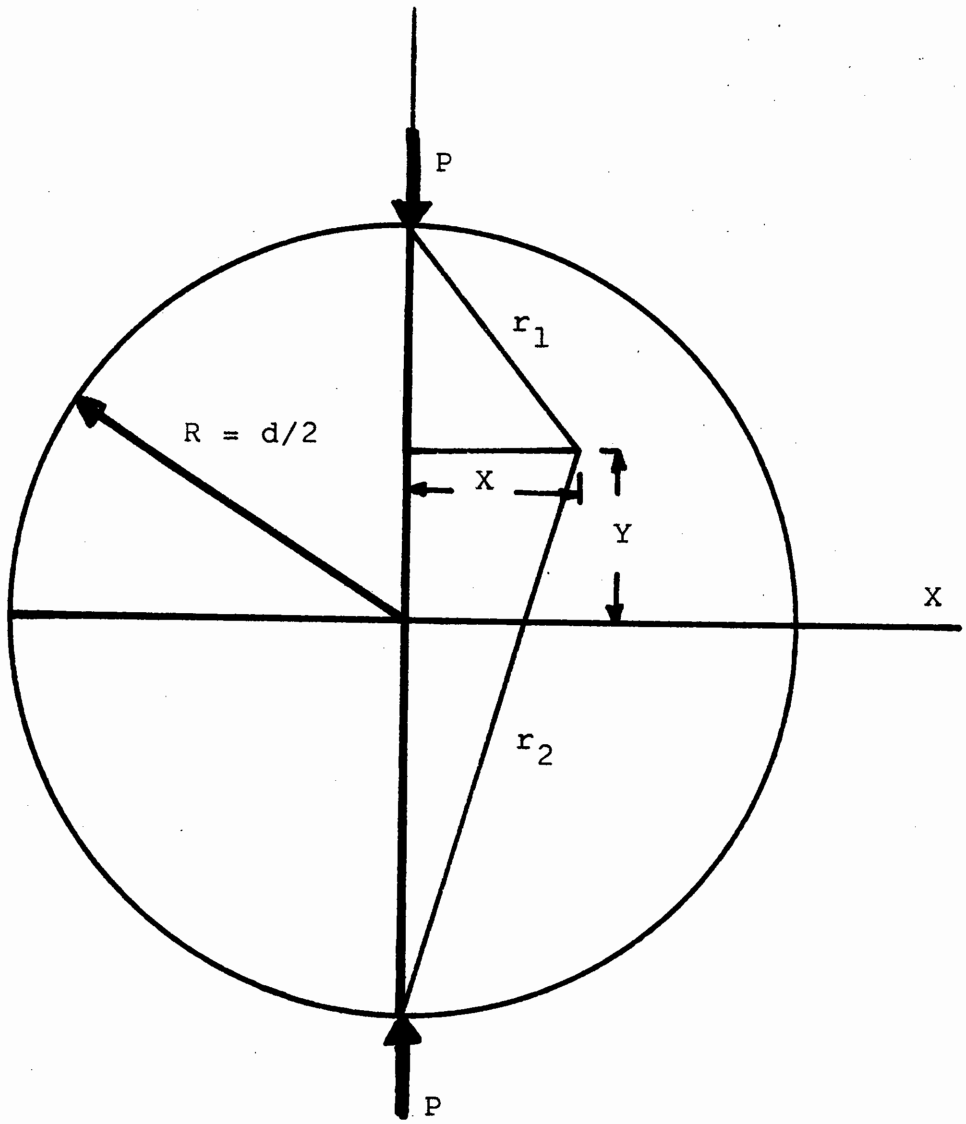


FIGURE 1. SPLIT CYLINDER TEST

UNIVERSAL TESTING MACHINE

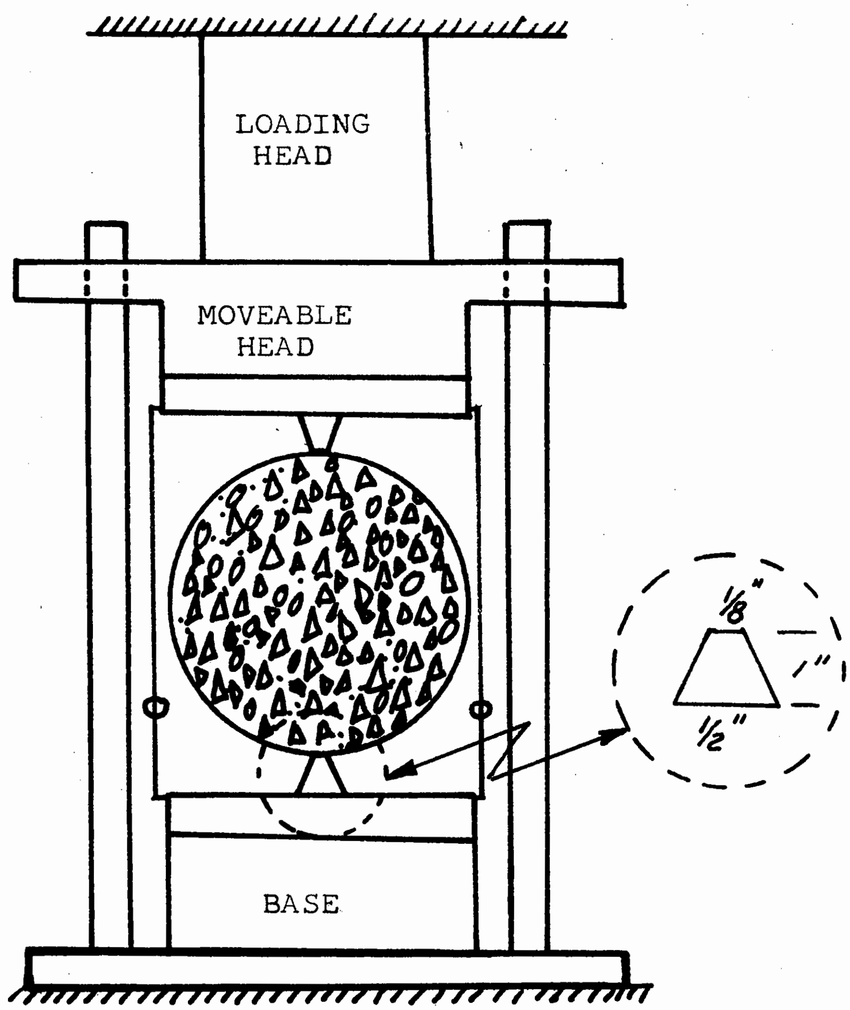


FIGURE 2. SPLIT CYLINDER DETAILS

They are:

$$(1) \quad S_x = -\frac{2P}{t}\,\frac{(R-Y)X^2}{r_1^{\,4}} + \frac{(R+Y)X^2}{r_2^{\,4}} - \frac{1}{d}$$

$$(2) \quad S_y = -\frac{2P}{t}\,\frac{(R-Y)^3}{r_1^{\,4}} + \frac{(R-Y)^3}{r_2^{\,4}} - \frac{1}{d}$$

$$(3) \quad V_{xy} = \frac{2P}{t}\,\frac{(R-Y)^2X}{r_1^{\,4}} - \frac{(R+Y)^2X}{r_2^{\,4}}$$

Where $S_x$ is the horizontal stress, $S_y$ is the vertical stress, $V_{xy}$ is the shear stress, d is the diameter of the specimen, R is the radius, t is the thickness of the disk perpendicular to the figure, P is the applied load, X, Y, r, and $r_2$ are the point coordinates as shown. By applying x = o and y = o to equations 1 and 2 they reduce to

$$(4) \quad S_y = -\frac{6P}{\pi td}$$

and

$$(5) \quad S_x = +\frac{2P}{\pi td}$$

$S_x$ is a principal tensile stress and it remains nearly constant over about 75% of the vertical diameter. In order for this test to be applied to a material, it must be at least three times stronger in compression than in tension. At low temperatures an asphaltic concrete meets this condition.

## MATERIALS & SAMPLE PREPARATION

The coarse and fine aggregate were from local glacial deposits. The aggregates were hand blended in the proportions stated in Table 1 and this gradation was held constant throughout all test series. 60-70 and 85-100 penetration grade asphalts from a Venezuelan crude were used. The asphalt properties are shown in Table 2.

The 1200 grams of aggregate were heated to 300°F and then blended with the asphalt. The percentage of asphalt was based on total weight of aggregate. The composite was mixed by means of a mechanical mixer for two minutes. It was then placed in a Marshall mold (4 in. diameter), hand rodded 25 times, and then compacted using the kneading compactor with a 500 p.s.i. foot pressure, a 0.5 second dwell time, and 50 blows on each end of the specimen. The sample was allowed to cool to room temperature, removed from the mold and stored at testing temperature for a period of four weeks. A series of ten specimens was made for each penetration asphalt, each asphalt content, and each test temperature.

At the end of this period the samples were weighed and the height of each determined. The average height (t) of all specimens was 2.5 inches. They were removed from the controlled temperature room in an insulated box and tested by means of the split cylinder using a loading rate of 6,000 lbs/minute. The vertical movement of the testing head was read at each 500 lb. load increment.

## RESULTS

The result of a typical split cylinder test is shown in

Table 1. AGGREGATE GRADATION

| Sieve Size | Percent Retained | Weight Used (Grams) |
|---|---|---|
| 1" | 0 | 0 |
| 3/4" | 6 | 72 |
| 1/2" | 14 | 96 |
| 3/8" | 20 | 72 |
| #4 | 50 | 360 |
| #10 | 65 | 180 |
| #20 | 73 | 96 |
| #40 | 79 | 72 |
| #80 | 87 | 96 |
| #200 | 95 | 96 |
| Pan | 100 | 60 |

Table 2. PROPERTIES OF ASPHALT

| Grade | 85-100 | 60-70 |
|---|---|---|
| Penetration, 100 g, 5 sec. 77F | 92 | 62 |
| Specific Gravity at 77F | 1.032 | 1.027 |
| Flash Point C.O.C. F | 570 | 585 |
| Loss on Heating 50 g, 5 hr @ 325F % | 0.12 | 0.08 |
| Penetration of Residue, % of original | 86 | 89 |
| Solubility in $CCL_4$% | 99.9 | 99.9 |
| Ductility at 77F 5 cm/min cm | 100+ | 100+ |

Figure 3. The motion of the loading head has been plotted along the abscissa and the force along the ordinate axis. The area under the curve to failure is the work required to fracture the specimen. The results of all the tests are shown in Appendix A with the accompanying statistical analysis. The work to failure is independent of the question of elastic or visco-elastic behavior. Computation of stresses has been avoided due to the uncertainty about the degree of elastic behavior. Each group of 10 samples resulted in a distribution of ultimate load values similar to those shown in Figure 4. Owing to the similarity of the curves for each test temperature, the force per unit of sample height has been shown for only one asphalt content in Figures 5 and 6. The similarity of behavior for the two grades of asphalt is readily apparent with the force per unit length always being lower for the 85-100 penetration grade asphalt.

It is recognized that the results of the split cylinder test on plastic materials could be influenced by the penetration of the loading edge into the sample. At 40$^{o}$F some penetration did occur, but at lower temperatures there was little, if any, indentation.

The work required to fracture the specimen as a function of temperature is shown in Figures 7 and 8. Regardless of penetration grade asphalt used or the asphalt content, the work required for failure always decreases with decreasing temperature. At the lower temperatures, the effect of asphalt content on work is minor, as seen in Figures 9 and 10 (see

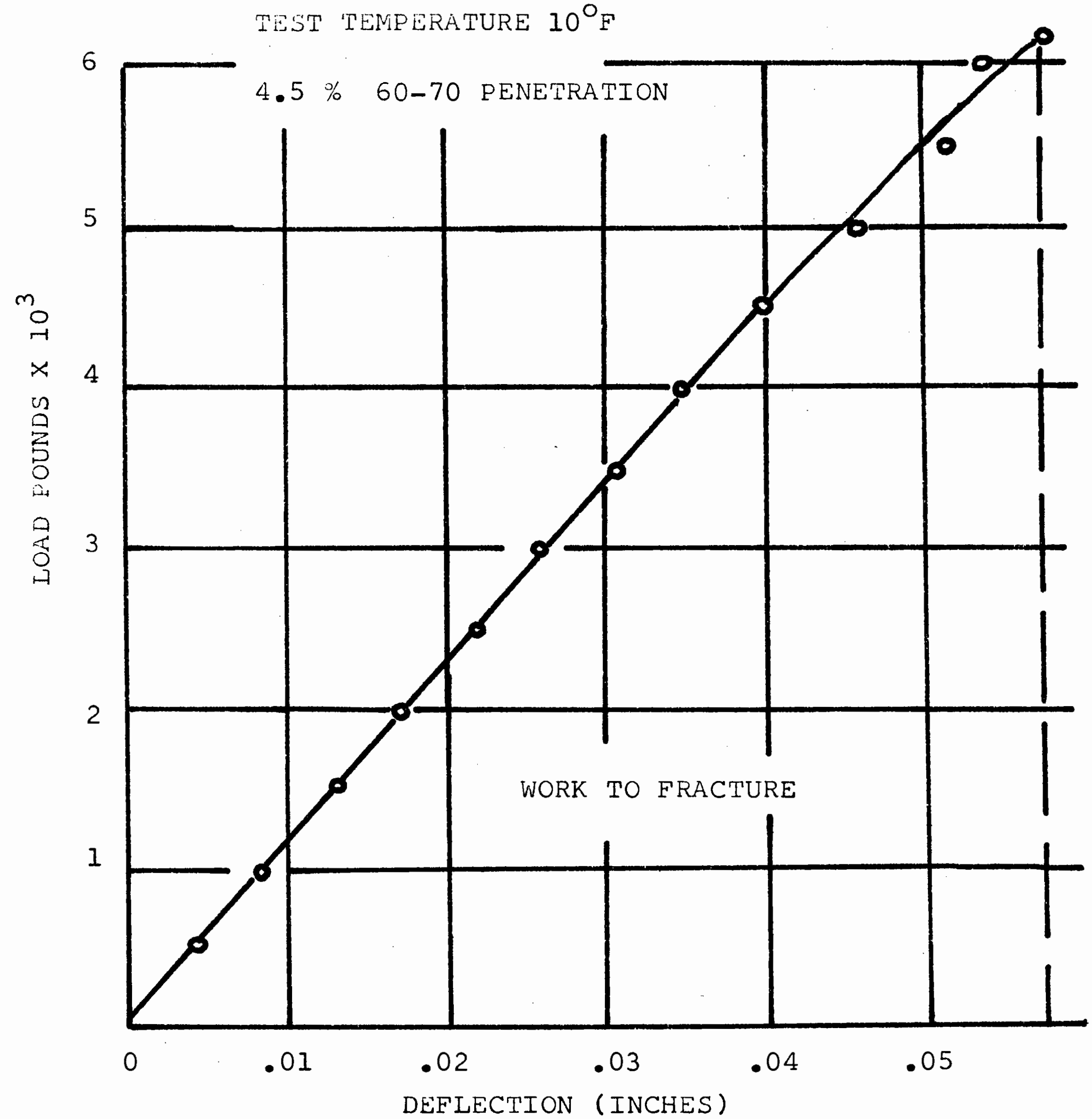


FIGURE 3. TYPICAL LOAD DEFLECTION CURVE FOR DETERMINING WORK TO FRACTURE SPLIT CYLINDER

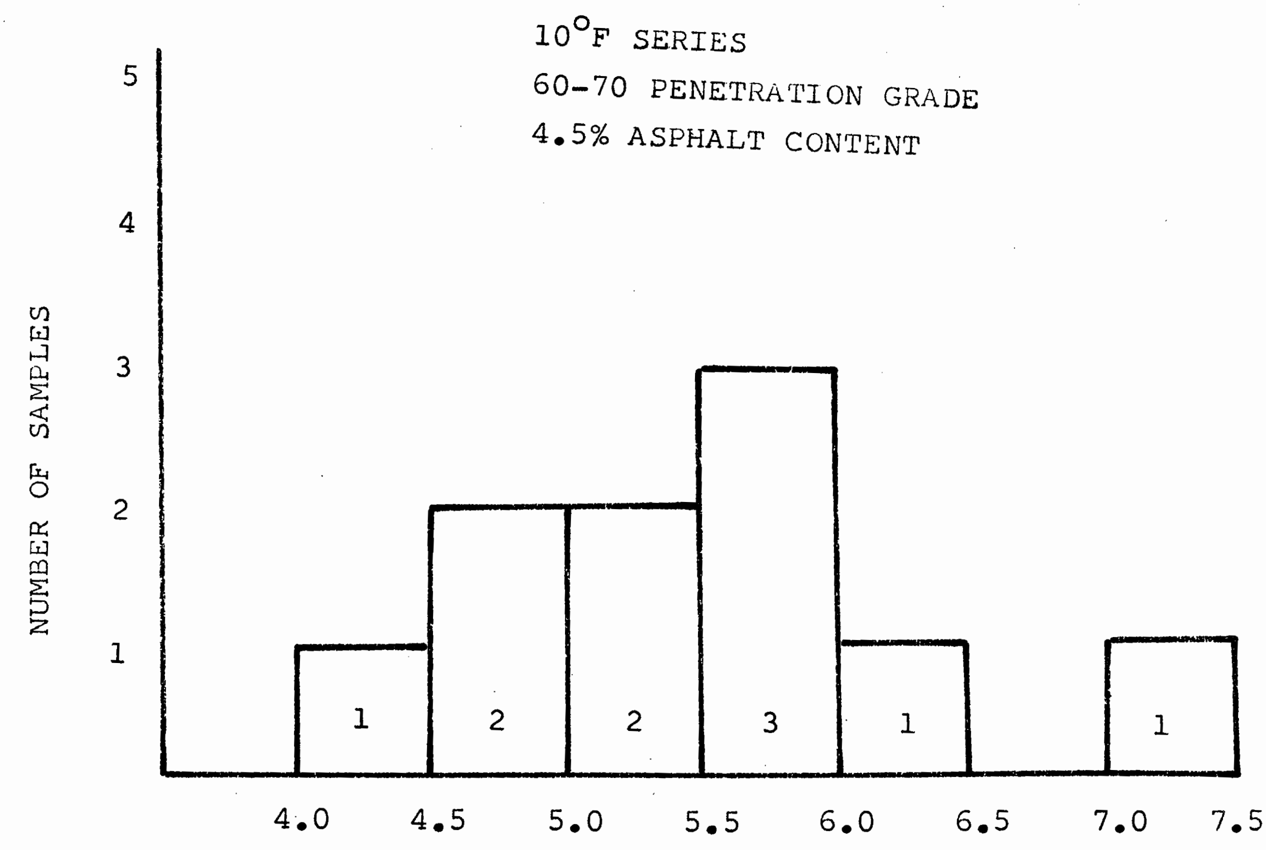


FIGURE 4. TYPICAL HISTOGRAM OF ULTIMATE LOAD VALUES

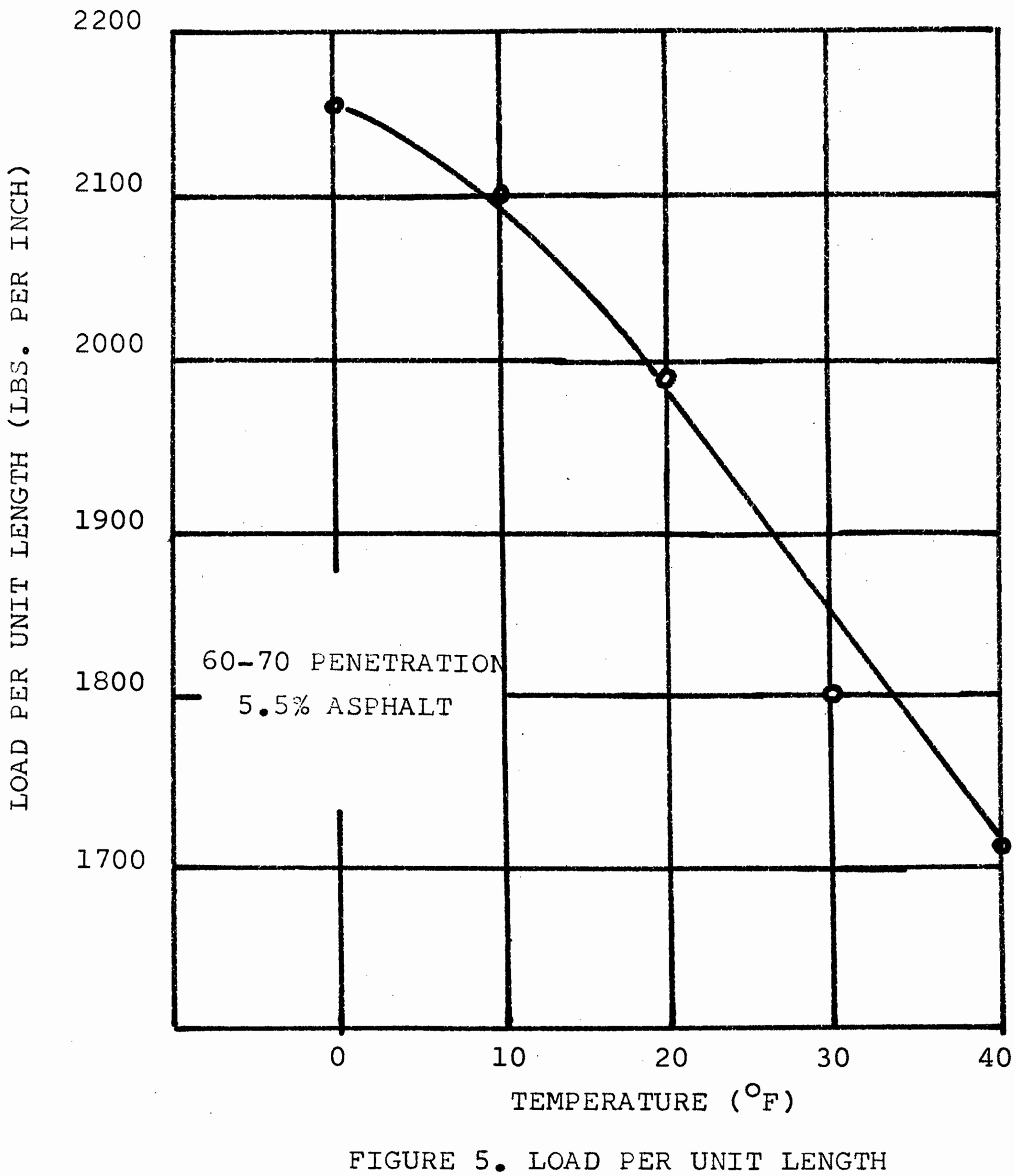


FIGURE 5. LOAD PER UNIT LENGTH VERSUS TEMPERATURE

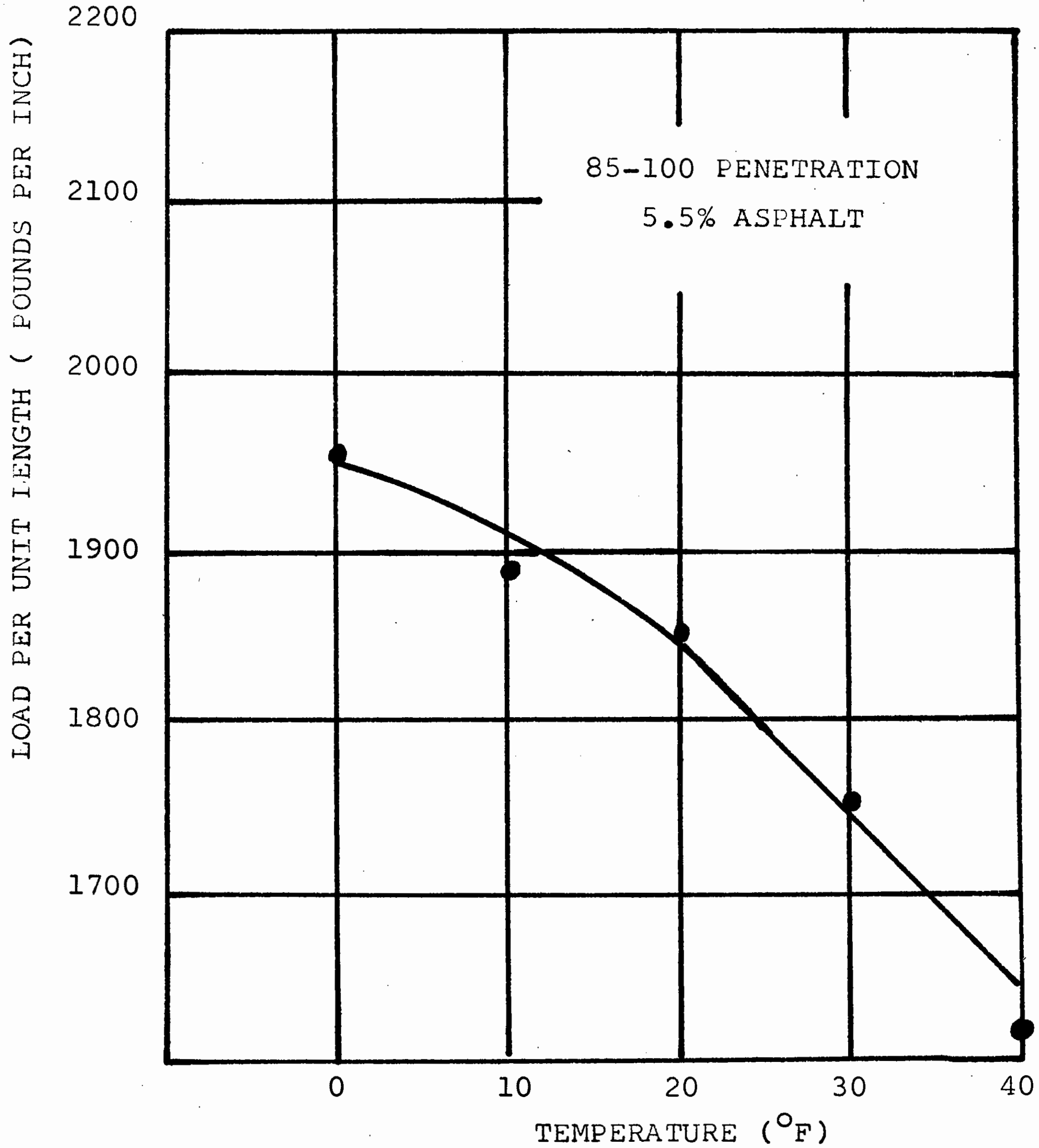


FIGURE 6. LOAD PER UNIT LENGTH VERSUS TEMPERATURE

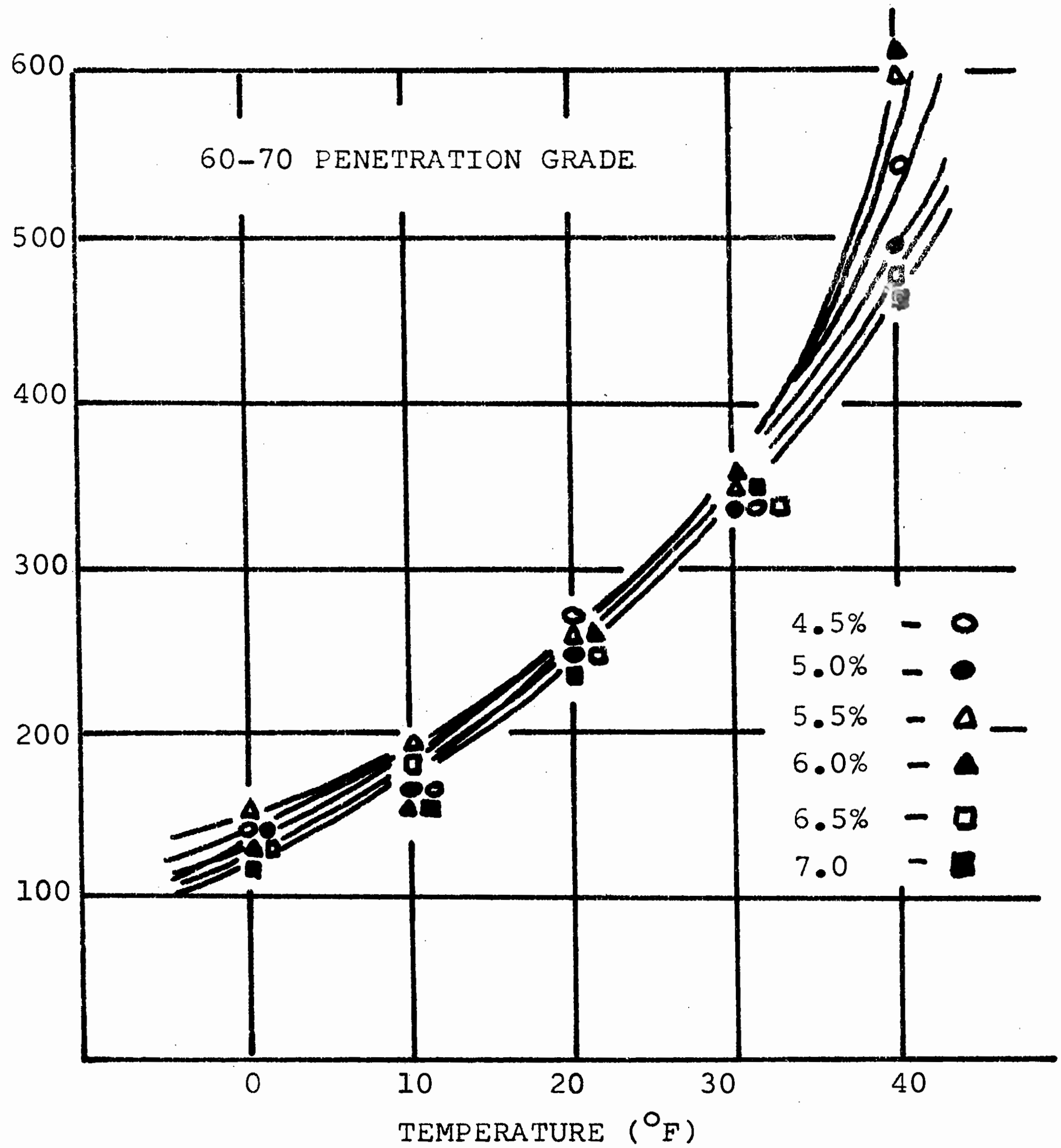


FIGURE 7. WORK VS. TEMPERATURE WITH VARIOUS ASPHALT CONTENTS

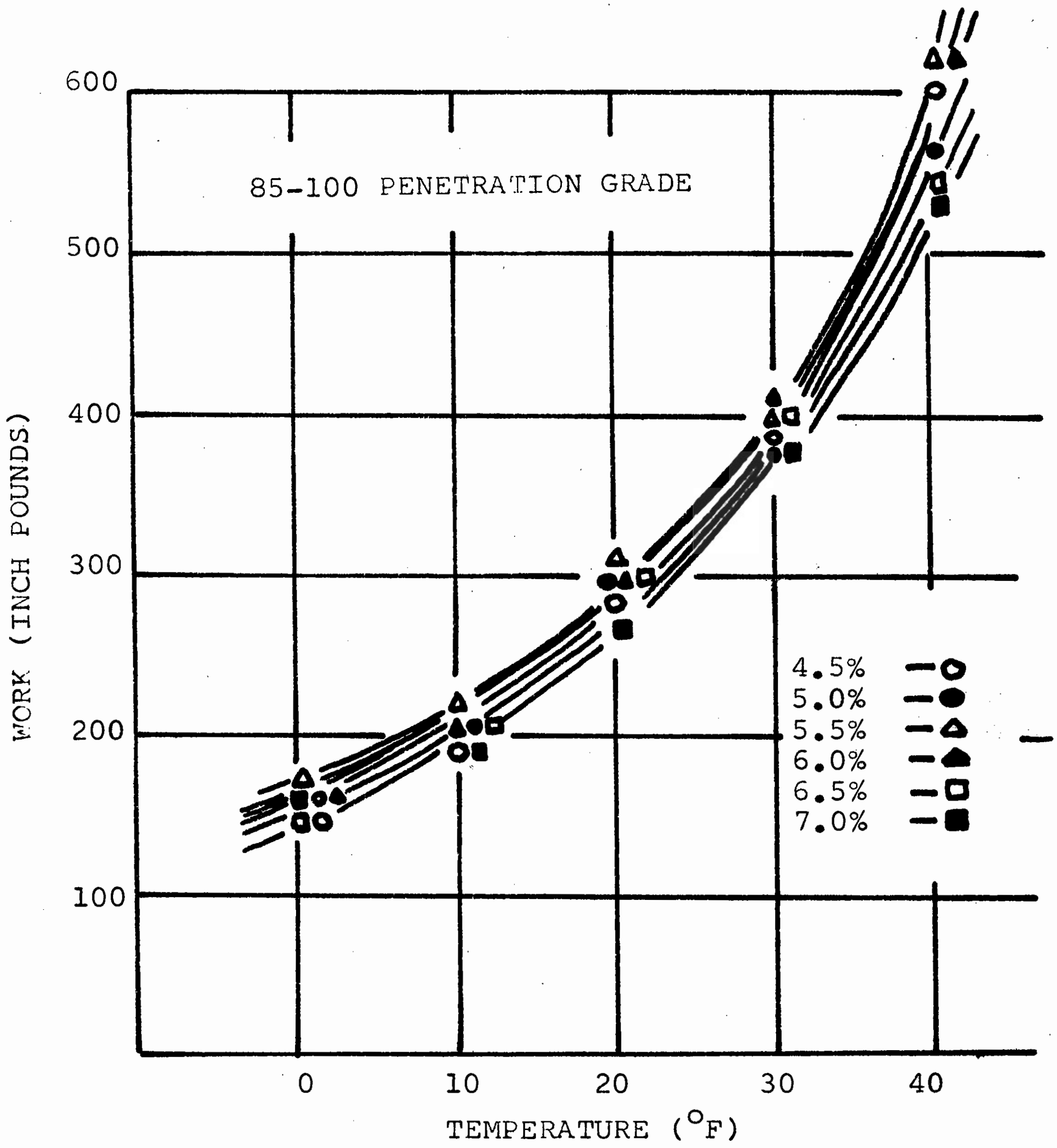


FIGURE 8. WORK VS. TEMPERATURE WITH VARIOUS ASPHALT CONTENTS

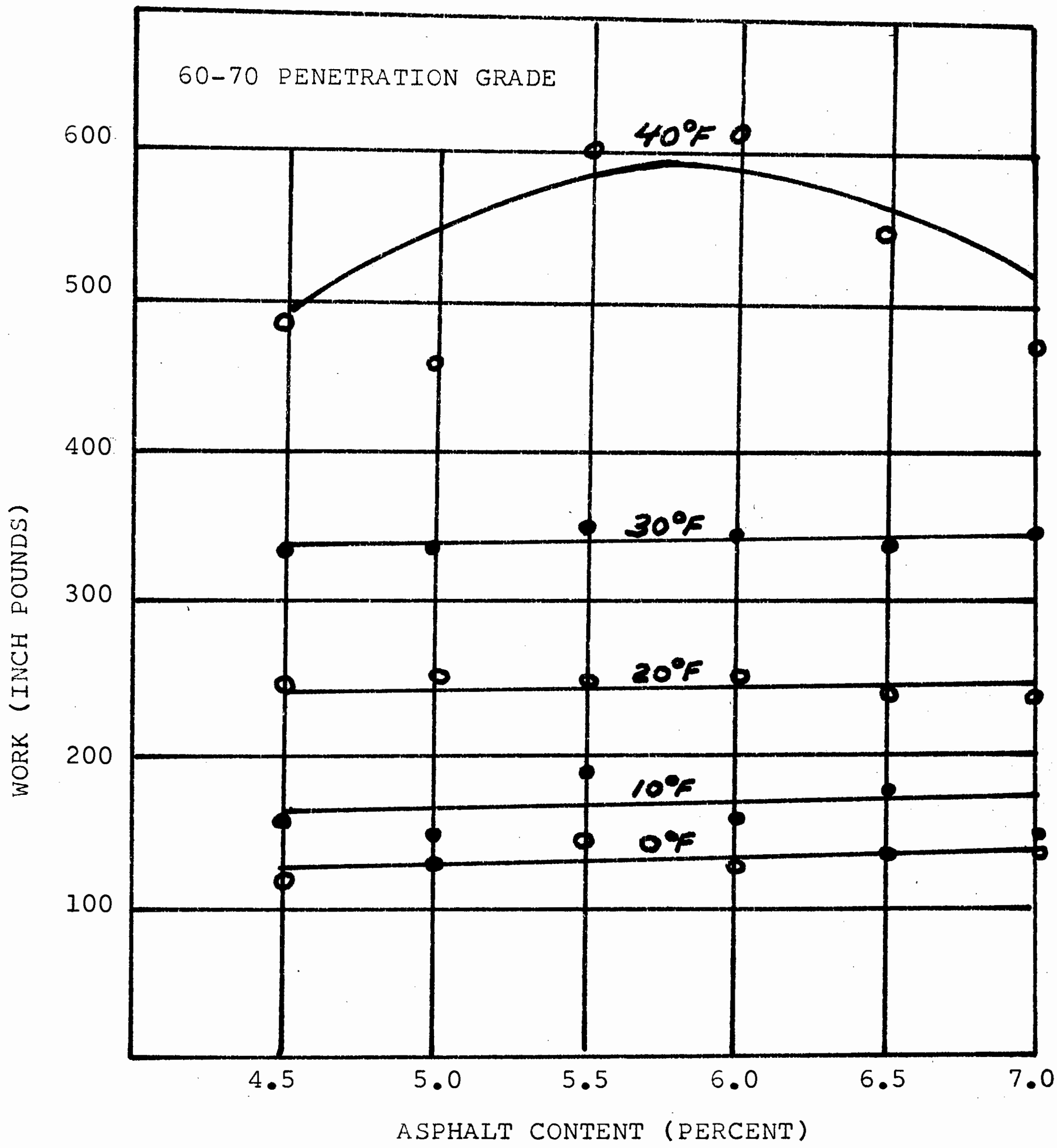


FIGURE 9. WORK VS. ASPHALT CONTENT
WITH VARIOUS TEMPERATURES

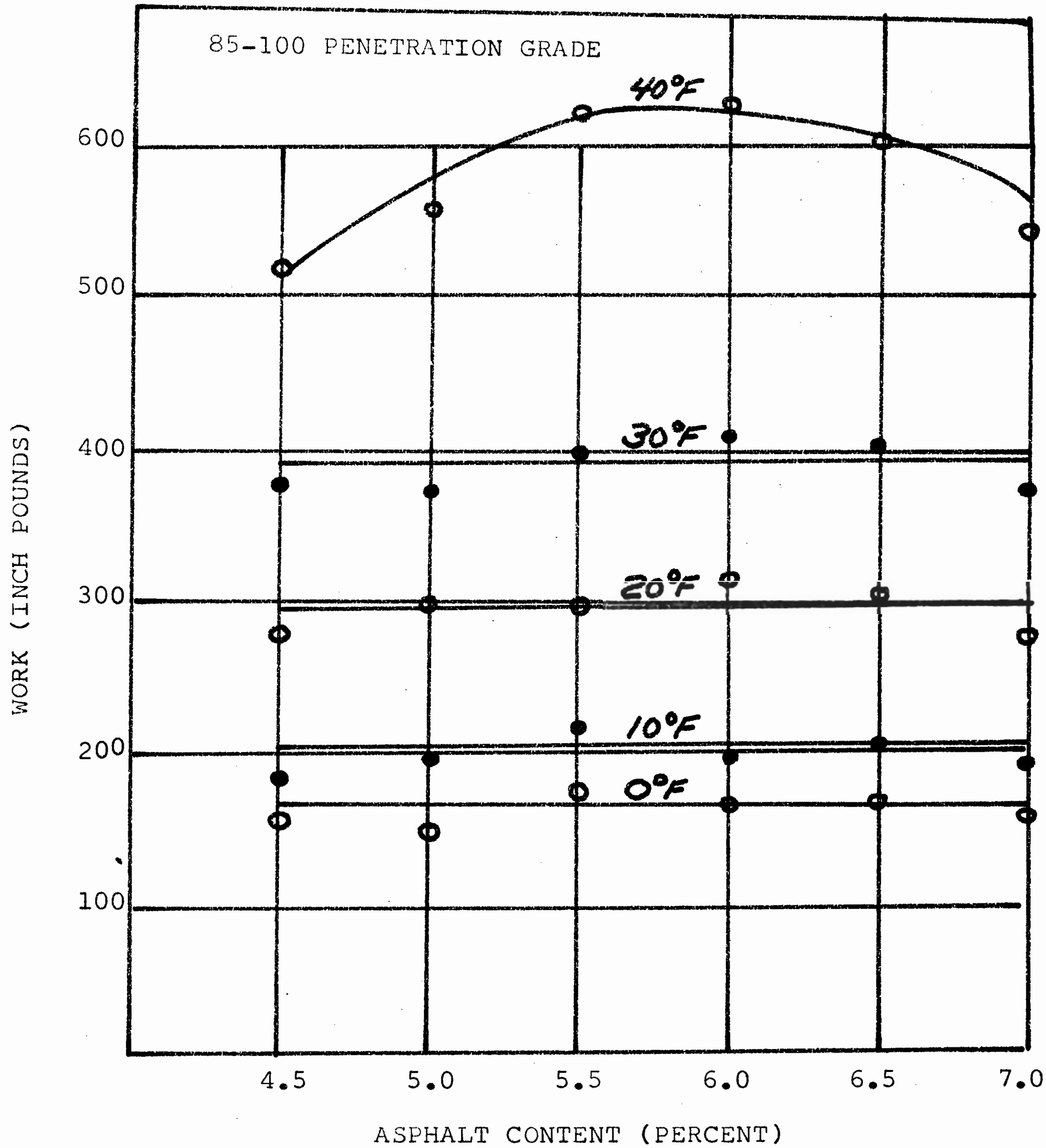


FIGURE 10. WORK VS. ASPHALT CONTENT WITH VARIOUS TEMPERATURES

Appendix A and B). At all temperatures, the load required to fracture the specimen was always greater for the 60-70 penetration grade asphalt. However, due to greater deformation, the work to fracture the specimen was greater for the 85-100 penetration grade asphalt.

## THE FATIGUE TEST

Whenever any material is subjected to a force, it resists the force by developing stresses within the material. If a given stress level is applied a sufficient number of times, failure will occur even though the applied stress is well below the ultimate strength of the material. A recent report by Vesic and Domaschuk (18) on the AASHO Road Test indicates that during periods of low temperature, there is a reduction in the value of the subgrade stress for a given load. This redistribution of stresses through the layered system could be caused by the fact that at low temperatures a bituminous pavement increases in stiffness and behaves in a semi-rigid manner. A pavement behaving in this manner tends to resist applied wheel loads by bending and, therefore, distributes the wheel load over a larger area of the subgrade. Repetitions of tensile stresses can ultimately lead to failure. Pioneering work by Griffith (19) indicated that cyclic tensile stresses that are below the ultimate tensile strength of a material can lead to cracking. This is especially true of brittle materials. As the pavement temperature decreases and the bituminous concrete behaves more nearly as a brittle solid, the susceptibility to fatigue becomes increasingly important. Flexible pavement design methods do not incorporate the concept of slab action, however, the split cylinder test results indicate that the material becomes rigid as the environmental temperature decreases. It is hypothesized that the behavior of the flexible pavement system is a combination of Boussinesq behavior and slab action. During the spring period flexible pavements are subject to what is termed "spring breakup". During this period the subgrade usually has a reduced strength and surface cracking could be caused by subgrade consolidation. On the other hand, the spring breakup cracks may be the first appearance of cracks formed during the winter.

Fatigue studies have been conducted in recent years by Monismith (20), Pell (21) and Jimenez and Galloway (22). Results from these studies indicates that bituminous material is subject to fatigue. These tests have been made on asphalt and mastics and over the higher ranges of temperature ($70^{\circ}F$ to $140^{\circ}F$). Pell conducted tests on asphalt and the results of these tests indicate that at low temperatures the fatigue characteristics are strain susceptible. That is, the number of cycles of a given strain that an asphalt can withstand decreases with decreasing temperature. The absolute number of these cycles were determined for asphalt alone and cannot be extended to an asphaltic concrete because the inclusion of aggregate creates a much different material. Also, Pell's test was a torsion test and although tensile failures were observed, the failures in the field appear to be direct tensile failures. Monismith subjected asphaltic concrete slabs to repeated vertical stress. He concluded that the fatigue life of a pavement subject to a given vertical stress increases with decreasing temperature. The range of applied stress was much lower than that encountered in the field and his conclusions were based on temperatures above 50$^{o}$F.

In order to evaluate the effects of temperature and curvature (degree of bend) on the fatigue life of asphaltic concrete, the testing device shown in Figures (11) and (12) was built. At a future date, slab sections can be removed from recently constructed pavements, and tested in the laboratory. Also, with refinements in measuring techniques, it may be possible to obtain field information on the relationship between wheel load, curvature and temperature. As of this time, there is little information available on curvature measurements.

The testing machine consists of a series of lower steel plates machined to a given radius of curvature and a flat upper plate that is placed in a rigid box. A one inch thick bituminous specimen was placed between the flat and curved plate so that it was tangent to the circular plate at point A. The specimen extended one inch beyond the end of the testing device permitting the attachment of a rod, which moved the end of the specimen up and down bending the material to the curvature of the plates. The rod was attached to an eccentric

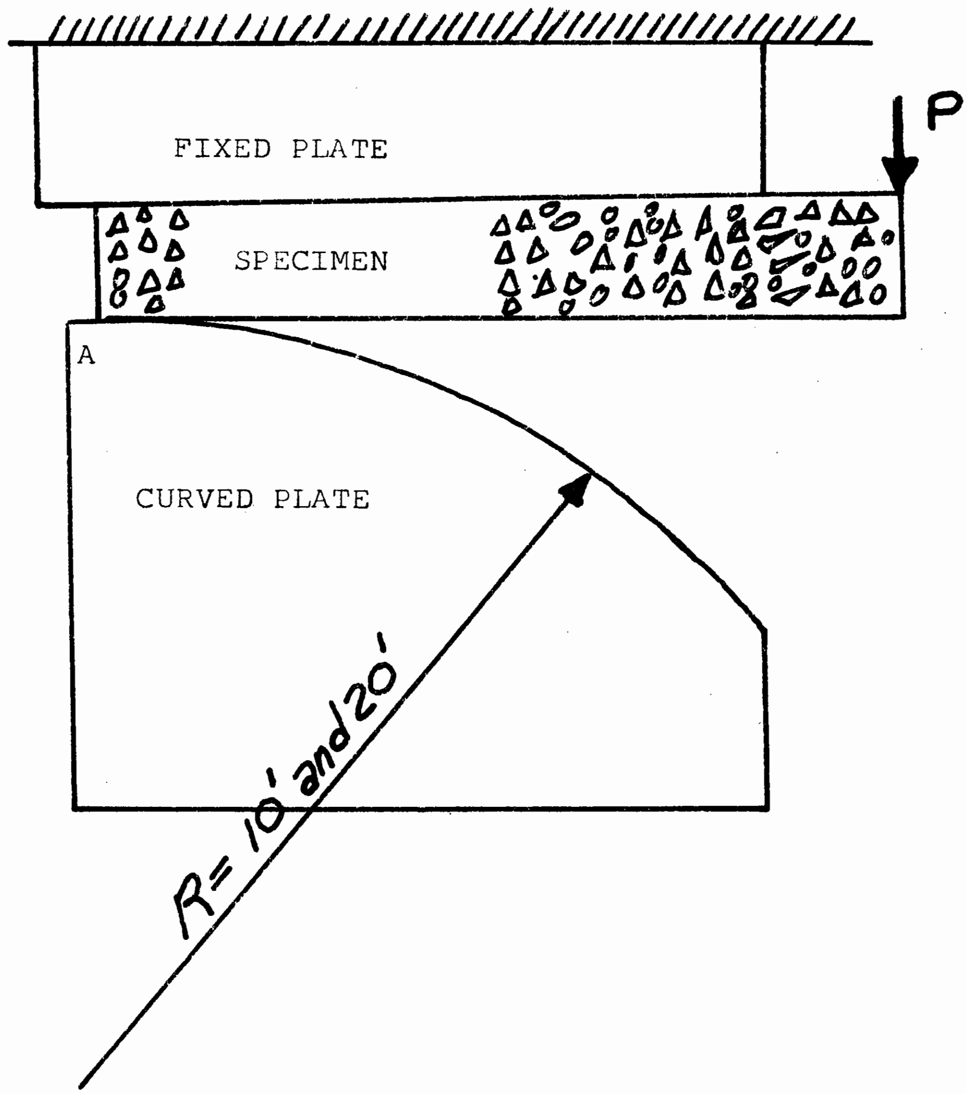


FIGURE 11. TESTING MACHINE

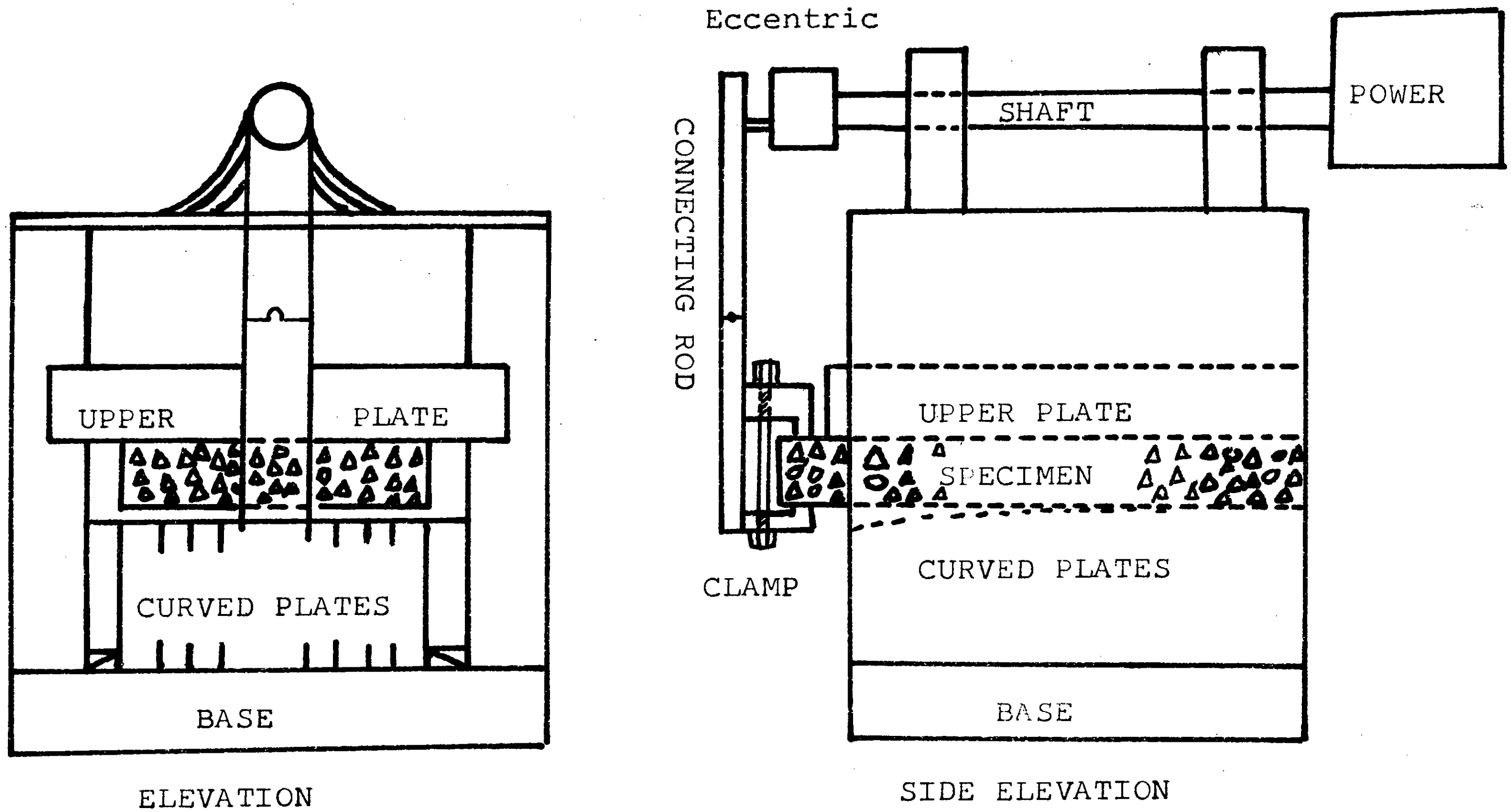


FIGURE 12. TESTING MACHINE DETAILS

shaft which was driven by a one-third horsepower gear head motor at an angular speed of 120 revolutions per minute.

The entire assembly was placed in a controlled temperature room and the specimen tested. The mix gradation and asphalt properties are shown in Tables 3 and 4. Only one asphalt content was tested, that corresponding to maximum Marshall Stability for the aggregate used.

The samples were prepared by mixing the coarse and fine aggregates in a mechanical mixer for two minutes, then molding it in three layers (50 tamps/layer) in a one inch by four inch by six inch rectangular mold as shown in Figure (13). The mold was then placed in a universal testing machine and a leveling load of 10,000 lbs. applied. The sample was removed from the mold, stored at test temperature for 5 days and then tested in repeated flexure.

A number of trials were made to develop a criteria for failure. A strain gage attached to the specimen which was continually monitored was not successful because it was impossible to predict where the initial crack would start. It was decided that the best criteria for failure was visual examination. At each test temperature and each radius of curvature it was possible to set up an examination schedule after a month of trials. With the ten foot radius plate the specimen was examined every fifteen minutes (1700 cycles). All failures, when using this plate, took place in less than eight hours. With the twenty foot radius plate, a visual examination was made every twelve hours (80,000 cycles),

Table 3. AGGREGATE GRADATION

| Sieve Size | Percent Retained | Weight Used (Grams) |
|---|---|---|
| 1" | 0 | 0 |
| 3/4" | 0 | 0 |
| 1/2" | 9 | 108 |
| 3/8" | 15 | 72 |
| #4 | 46 | 372 |
| #10 | 63 | 204 |
| #20 | 71 | 96 |
| #40 | 77 | 72 |
| #80 | 86 | 108 |
| #200 | 95 | 108 |
| Pan | 100 | 60 |

ONE-HALF INCH STEEL PLATE MOLD

INSIDE DIMENSIONS ONE INCH BY FOUR INCHES BY SIX INCHES

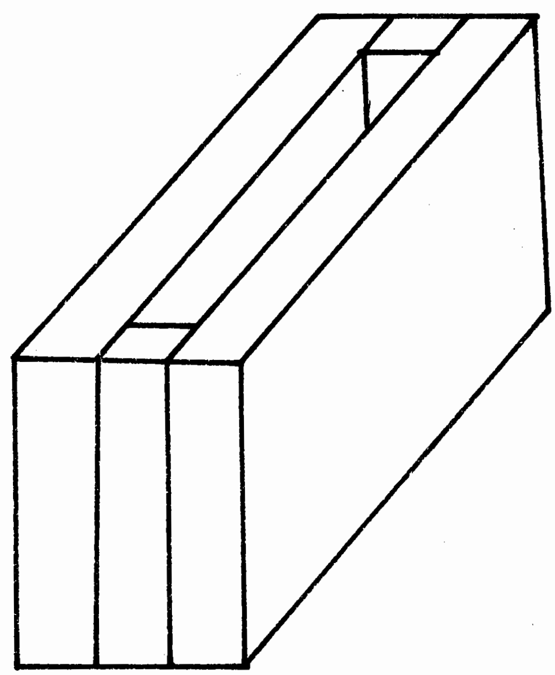

FIGURE 13. MOLD FOR FATIGUE SPECIMENS

because the minimum time for failure was around two days (432,000 cycles).

A series of ten specimens were tested at each temperature ($0^{\circ}$ to $40^{\circ}F$) and curvature (10 foot and 20 foot radius).

RESULTS

The results of the tests are shown in Figures 14 and 15, for the ten and twenty foot radius plate. The complete test results are shown in Appendix B. With the ten foot radius plate the number of cycles to failure decreased with decreasing temperature from an average of 32,000 cycles at $40^{\circ}F$ to 4,760 cycles at $0^{\circ}F$. From $20^{\circ}F$ to $0^{\circ}F$ the shape of the histogram is almost the same. Above $20^{\circ}F$ the fatigue life of the specimen increased. There is a great deal of scatter in the data, which is also evident in the field where two adjacent sections of a lane will have a crack in one and none in the next section.

The results of the twenty foot radius plates showed no failure at $40^{\circ}F$ and $30^{\circ}F$. Five specimens were tested at each of these temperatures and no cracking was visible. From $20^{\circ}F$ to $0^{\circ}F$ cracking did occur in all specimens. This crack usually occurred across the middle third of the specimen. See Figure 16. The data in Appendix B indicates that at each test temperature there is no difference in the fatigue life of the specimens as a function of temperature when using the twenty foot circular plates.

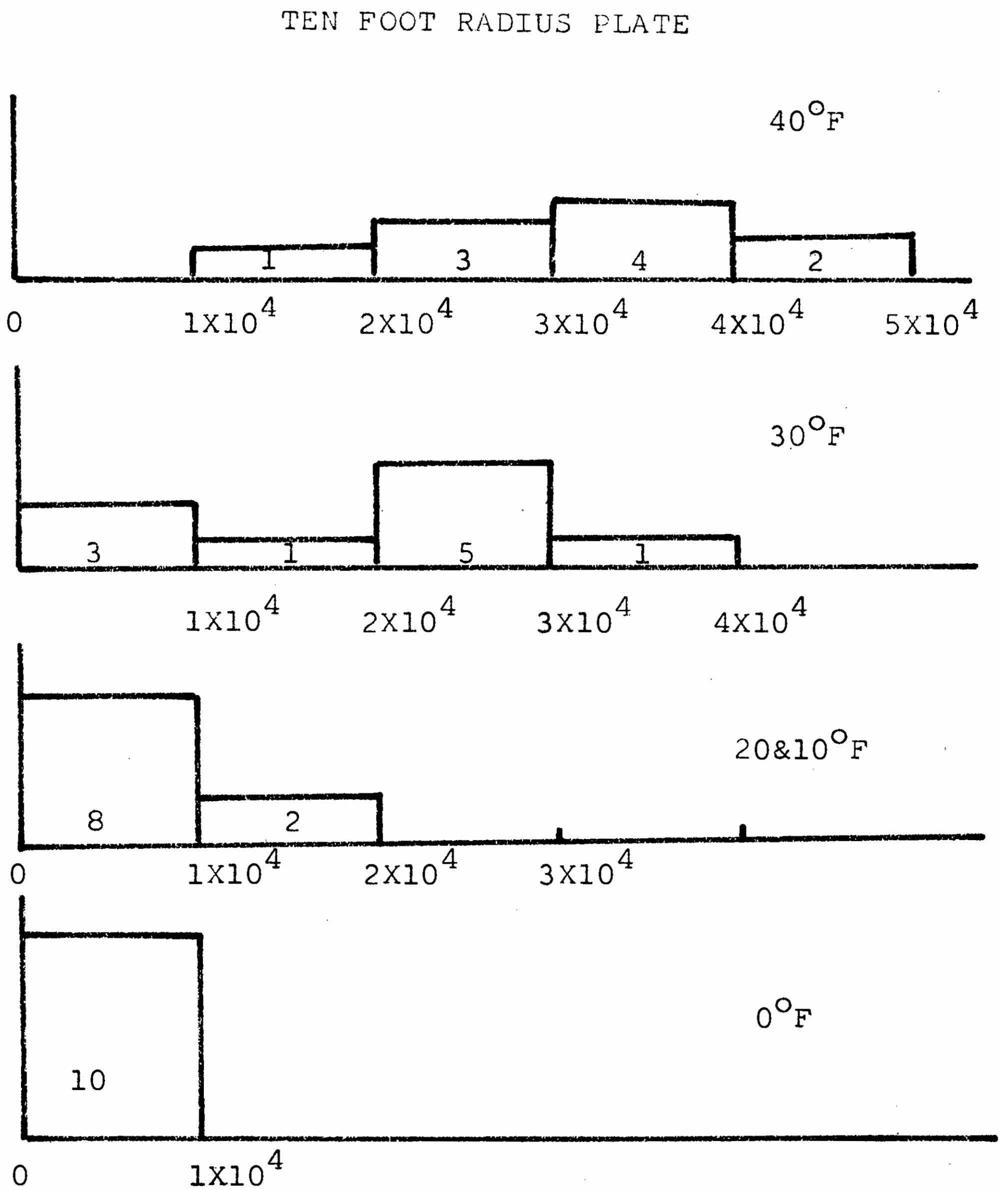


FIGURE 14. HISTOGRAM CYCLES TO FAILURE VERSUS TEMPERATURE

TWENTY FOOT RADIUS PLATE

NO FAILURES WERE OBSERVED AT 30&40°F

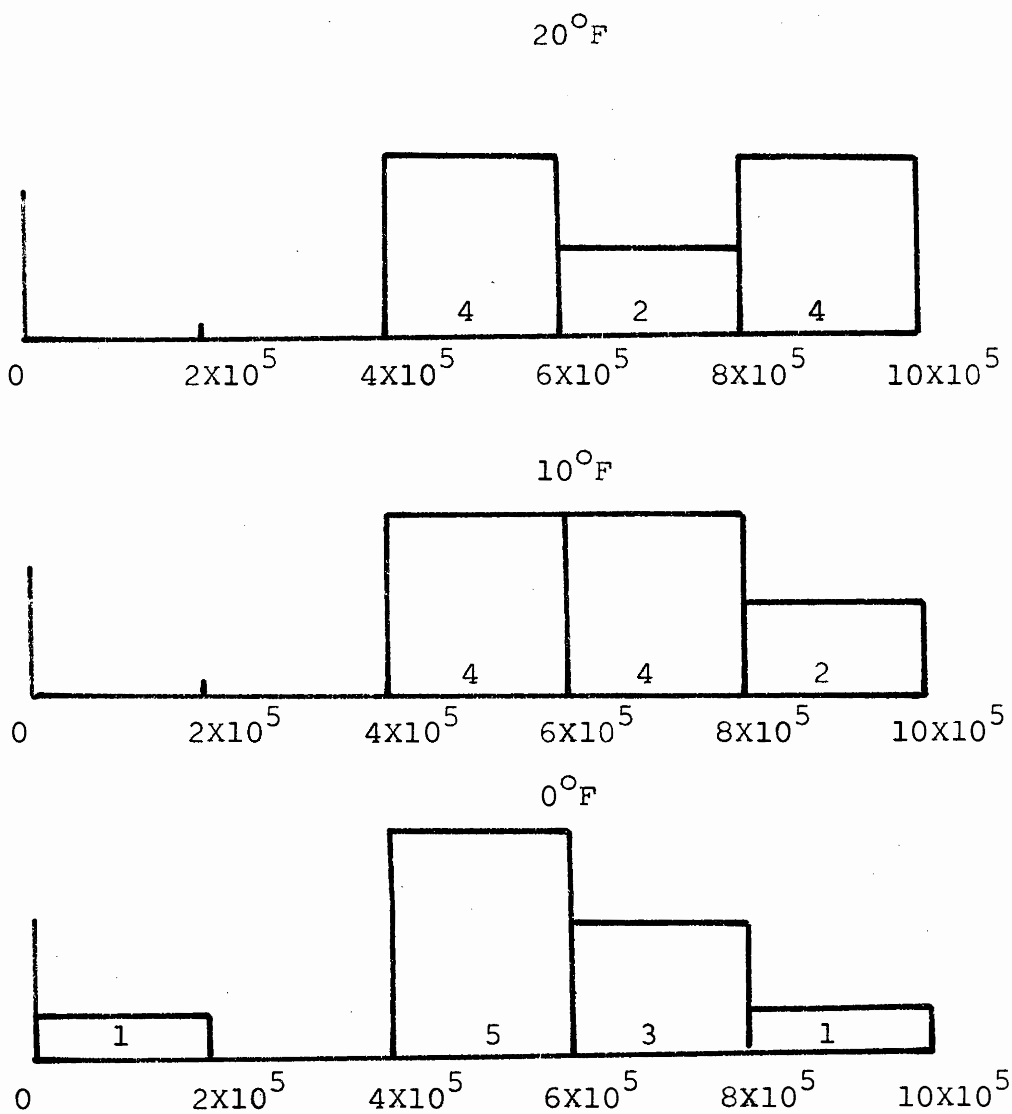


FIGURE 15. HISTOGRAM CYCLES TO FAILURE VERSUS TEMPERATURE

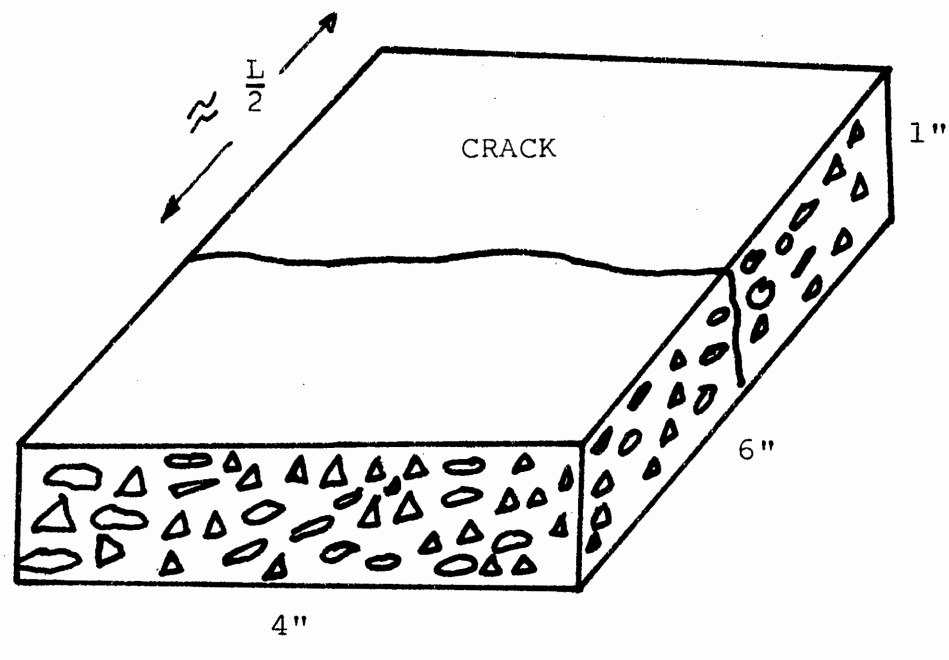


FIGURE 16. TYPICAL CRACK FAILURE

## TEST APPLICATIONS

Longitudinal cracking of asphaltic concrete can be caused by many things. The subgrade or base may fail and this could be reflected through the surface course. Heavy wheel load repetitions by continually flexing the pavement could lead to fatigue failure. The tensile and fatigue properties of asphaltic concrete are temperature susceptible as the test data indicates. With decreasing temperature asphalt undergoes a change in state from ductile to brittle behavior. The temperature at which this occurs is not uniquely defined, but rather consists of a zone from around 20$^{\circ}$F to 50$^{\circ}$F and undoubtedly is a function of asphalt source and penetration grade. At low temperatures the material is stronger in a strength sense, that is it takes a larger stress to fracture the binder but the strain at failure is reduced so that the mode of failure is brittle fracture rather than yielding, and the fatigue strength is lowered. Flexible pavements are not designed to resist bending moment and yet at lower temperatures slab action develops. Design methods usually require a minimum of 2 percent air voids in a bituminous mix in order to prevent bleeding of the asphalt. These same voids at 20$^{\circ}$F can cause stress concentrations when the material is in a brittle state, so that the actual stress within the material may be higher than that predicted by the equations of elasticity. Beam action requires that the pavement withstand tension which can lead to fatigue failure.

The tests that are described herein can be applied to specimments for comparing the effect of asphalt content, compactive effort and time on the low temperature pavement characteristics.

Two problems that are extremely difficult to evaluate are the actual radius of curvature that a given wheel load causes and the effect of aging of the asphalt binder with time. Deflection studies do not determine the radius of curvature with any degree of accuracy and by attempting to plot a curve through the pavement profile, the radius of curvature appears to be small, but its absolute value is indeterminate. Asphalt decreases in ductility with time which could tend to reduce the fatigue life. A pavement that is new may exhibit good fatigue characteristics during the first few years, but the combination of loading history plus decreased ductility could lead to fatigue failure after three or four years.

## RECOMMENDED RESEARCH

(1) It is recommended that the fatigue machine be used to evaluate existing highway pavements.

(2) A test road could be built to determine the interrelationship between method of construction, asphalt content and the effect of time on the fatigue and low temperature tensile properties of actual field samples.

# APPENDIX A

# SPLIT CYLINDER TEST RESULTS

ASPHALT CONTENT (Percent)

| 4.5 | 5.0 | 5.5 | 6.0 | 6.5 | 7.0 | |
|---|---|---|---|---|---|---|
| 90 | 110 | 110 | 130 | 100 | 160 | |
| 100 | 100 | 140 | 100 | 130 | 120 | |
| 150 | 170 | 190 | 190 | 130 | 100 | |
| 110 | 110 | 130 | 110 | 130 | 120 | |
| 100 | 120 | 120 | 110 | 140 | 120 | |
| 100 | 120 | 160 | 160 | 180 | 140 | |
| 140 | 140 | 160 | 140 | 150 | 120 | |
| 120 | 120 | 140 | 130 | 160 | 100 | |
| 80 | 100 | 120 | 110 | 140 | 110 | |
| 110 | 120 | 140 | 130 | 140 | 120 | mean |

Work (Inch-Pounds) 60-70 penetration at $0^{\circ}F$

ASPHALT CONTENT (Percent)

| 4.5 | 5.0 | 5.5 | 6.0 | 6.5 | 7.0 | |
|---|---|---|---|---|---|---|
| 180 | 130 | 200 | 180 | 120 | 190 | |
| 200 | 210 | 120 | 190 | 180 | 180 | |
| 130 | 150 | 180 | 100 | 220 | 110 | |
| 140 | 140 | 230 | 140 | 200 | 140 | |
| 160 | 140 | 200 | 140 | 180 | 140 | |
| 150 | 120 | 170 | 150 | 150 | 140 | |
| 110 | 160 | 180 | 130 | 160 | 130 | |
| 180 | 120 | 180 | 160 | 200 | 170 | |
| 200 | 100 | 200 | 150 | 190 | 140 | |
| 150 | 130 | 240 | 160 | 200 | 160 | |
| 160 | 140 | 190 | 150 | 180 | 150 | mean |

Work (inch-Pounds) 60-70 penetration at $10^{\circ}F$

ASPHALT CONTENT (Percent)

| 4.5 | 5.0 | 5.5 | 6.0 | 6.5 | 7.0 | |
|---|---|---|---|---|---|---|
| 290 | 210 | 250 | 230 | 250 | 280 | |
| 210 | 270 | 270 | 260 | 260 | 220 | |
| 250 | 260 | 280 | 250 | 240 | 240 | |
| 240 | 270 | 210 | 290 | 200 | 210 | |
| 230 | 260 | 290 | 280 | 230 | 250 | |
| 270 | 300 | 230 | 230 | 210 | 270 | |
| 220 | 220 | 210 | 250 | 240 | 250 | |
| 230 | 240 | 250 | 230 | 250 | 260 | |
| 280 | 260 | 220 | 270 | 260 | 240 | |
| 280 | 310 | 290 | 210 | 260 | 280 | |
| 250 | 260 | 250 | 250 | 240 | 250 | mean |

Work (Inch-Pounds) 60-70 penetration at $20^{\circ}F$

ASPHALT CONTENT (Percent)

| 4.5 | 5.0 | 5.5 | 6.0 | 6.5 | 7.0 | |
|---|---|---|---|---|---|---|
| 290 | 280 | 390 | 350 | 350 | 300 | |
| 290 | 300 | 320 | 300 | 290 | 390 | |
| 350 | 300 | 340 | 350 | 320 | 350 | |
| 320 | 330 | 320 | 390 | 340 | 360 | |
| 360 | 350 | 320 | 400 | 360 | 310 | |
| 370 | 370 | 370 | 330 | 330 | 310 | |
| 360 | 350 | 380 | 300 | 370 | 340 | |
| 340 | 340 | 330 | 370 | 340 | 330 | |
| 340 | 390 | 360 | 360 | 290 | 350 | |
| 280 | 290 | 370 | 350 | 310 | 360 | |
| 330 | 330 | 350 | 350 | 330 | 340 | mean |

Work (Inch-Pounds) 60-70 penetration at 30°F

ASPHALT CONTENT (Percent)

| 4.5 | 5.0 | 5.5 | 6.0 | 6.5 | 7.0 | |
|---|---|---|---|---|---|---|
| 490 | 590 | 600 | 580 | 610 | 430 | |
| 460 | 450 | 590 | 610 | 510 | 500 | |
| 430 | 430 | 590 | 610 | 590 | 460 | |
| 510 | 510 | 630 | 650 | 540 | 480 | |
| 480 | 520 | 650 | 560 | 510 | 490 | |
| 520 | 440 | 570 | 560 | 500 | 480 | |
| 450 | 430 | 600 | 590 | 470 | 490 | |
| 490 | 530 | 610 | 620 | 490 | 510 | |
| 470 | 450 | 640 | 640 | 540 | 510 | |
| 500 | 450 | 620 | 580 | 540 | 450 | |
| 480 | 460 | 600 | 600 | 530 | 480 | mean |

Work (Inch-Pounds) 60-70 penetration at 40$^{o}$F

ASPHALT CONTENT (Percent)

| 4.5 | 5.0 | 5.5 | 6.0 | 6.5 | 7.0 | |
|---|---|---|---|---|---|---|
| 160 | 160 | 180 | 140 | 190 | 210 | |
| 180 | 140 | 180 | 200 | 150 | 200 | |
| 170 | 150 | 160 | 160 | 210 | 180 | |
| 160 | 160 | 190 | 170 | 180 | 130 | |
| 180 | 140 | 170 | 180 | 200 | 150 | |
| 170 | 170 | 200 | 170 | 150 | 160 | |
| 190 | 150 | 130 | 210 | 190 | 170 | |
| 130 | 130 | 170 | 110 | 180 | 140 | |
| 170 | 90 | 200 | 140 | 120 | 120 | |
| 90 | 110 | 120 | 120 | 130 | 140 | |
| 160 | 140 | 170 | 160 | 170 | 160 | mean |

Work (Inch-Pounds) 85-100 penetration at $0^{o}F$

ASPHALT CONTENT (Percent)

| 4.5 | 5.0 | 5.5 | 6.0 | 6.5 | 7.0 | |
|---|---|---|---|---|---|---|
| 220 | 200 | 270 | 250 | 200 | 190 | |
| 190 | 240 | 200 | 180 | 230 | 200 | |
| 190 | 190 | 220 | 210 | 210 | 180 | |
| 190 | 170 | 230 | 190 | 190 | 240 | |
| 220 | 220 | 180 | 150 | 200 | 220 | |
| 190 | 140 | 200 | 180 | 210 | 170 | |
| 220 | 210 | 210 | 210 | 180 | 150 | |
| 150 | 200 | 220 | 200 | 220 | 220 | |
| 210 | 230 | 260 | 220 | 150 | 240 | |
| 120 | 200 | 210 | 210 | 210 | 90 | |
| 190 | 200 | 220 | 200 | 200 | 190 | mean |

Work (Inch-Pounds) 85-100 penetration at $10^{o}F$

ASPHALT CONTENT (Percent)

| 4.5 | 5.0 | 5.5 | 6.0 | 6.5 | 7.0 | |
|---|---|---|---|---|---|---|
| 300 | 300 | 290 | 300 | 280 | 290 | |
| 280 | 350 | 270 | 350 | 320 | 240 | |
| 220 | 280 | 330 | 310 | 380 | 350 | |
| 280 | 320 | 310 | 370 | 280 | 310 | |
| 320 | 270 | 360 | 280 | 220 | 240 | |
| 270 | 240 | 240 | 250 | 320 | 250 | |
| 250 | 300 | 260 | 320 | 290 | 260 | |
| 280 | 300 | 330 | 280 | 310 | 320 | |
| 310 | 340 | 310 | 310 | 300 | 260 | |
| 290 | 300 | 300 | 330 | 300 | 280 | |
| 280 | 300 | 300 | 310 | 300 | 280 | mean |

Work (Inch-Pounds) 85-100 penetration at $20^{o}F$

ASPHALT CONTENT (Percent)

| 4.5 | 5.0 | 5.5 | 6.0 | 6.5 | 7.0 | |
|---|---|---|---|---|---|---|
| 410 | 390 | 450 | 450 | 400 | 330 | |
| 370 | 400 | 390 | 380 | 400 | 410 | |
| 420 | 460 | 390 | 390 | 380 | 360 | |
| 380 | 320 | 370 | 430 | 370 | 400 | |
| 340 | 340 | 430 | 390 | 440 | 380 | |
| 350 | 380 | 360 | 490 | 420 | 420 | |
| 380 | 370 | 410 | 430 | 380 | 370 | |
| 350 | 330 | 470 | 380 | 480 | 380 | |
| 440 | 370 | 360 | 370 | 380 | 400 | |
| 360 | 340 | 370 | 390 | 350 | 350 | |
| 380 | 370 | 400 | 410 | 400 | 380 | mean |

Work (Inch-Pounds) 85-100 penetration at 30$^{\circ}$F

ASPHALT CONTENT (Percent)

| 4.5 | 5.0 | 5.5 | 6.0 | 6.5 | 7.0 | |
|---|---|---|---|---|---|---|
| 530 | 580 | 580 | 600 | 560 | 530 | |
| 540 | 530 | 640 | 640 | 630 | 510 | |
| 470 | 510 | 570 | 570 | 610 | 590 | |
| 500 | 580 | 630 | 580 | 550 | 500 | |
| 480 | 480 | 580 | 640 | 580 | 560 | |
| 520 | 600 | 660 | 640 | 640 | 500 | |
| 540 | 580 | 620 | 630 | 640 | 590 | |
| 520 | 550 | 600 | 540 | 630 | 560 | |
| 490 | 530 | 560 | 580 | 540 | 580 | |
| 510 | 560 | 660 | 580 | 620 | 580 | |
| 510 | 550 | 610 | 600 | 600 | 550 | mean |

Work (Inch-Pounds) 85-100 penetration at 40$^{o}$F

APPENDIX B

FATIGUE TEST RESULTS

CYCLES TO FAILURE VERSUS TEMPERATURE, TEN FOOT RADIUS

| 40°F | 30°F | 20°F | 10°F | 0°F |
|---|---|---|---|---|
| 13,600 | 6,800 | 3,400 | 1,700 | 1,700 |
| 22,100 | 8,500 | 3,400 | 1,700 | 1,700 |
| 27,200 | 8,500 | 6,800 | 3,400 | 3,400 |
| 28,900 | 18,700 | 6,800 | 3,400 | 3,400 |
| 32,300 | 22,100 | 6,800 | 3,400 | 3,400 |
| 32,300 | 25,500 | 6,800 | 5,100 | 5,100 |
| 37,400 | 25,500 | 6,800 | 5,100 | 5,100 |
| 39,100 | 27,200 | 8,500 | 6,800 | 6,800 |
| 42,500 | 28,900 | 11,900 | 11,900 | 8,500 |
| 45,900 | 37,400 | 11,900 | 15,300 | 8,500 |

CYCLES TO FAILURE VERSUS TEMPERATURE, TWENTY FOOT RADIUS

| $40^{\circ}F$ | $30^{\circ}F$ | $20^{\circ}F$ | $10^{\circ}F$ | $0^{\circ}F$ |
|---|---|---|---|---|
| | | 432,000 | 432,000 | 172,800 |
| | | 432,000 | 432,000 | 432,000 |
| | | 432,000 | 432,000 | 432,000 |
| | | 518,400 | 432,000 | 432,000 |
| N O N E | N O N E | 691,200 | 604,800 | 432,000 |
| | | 777,000 | 604,800 | 518,400 |
| | | 864,000 | 604,800 | 604,800 |
| | | 864,000 | 777,000 | 604,800 |
| | | 950,400 | 864,000 | 691,200 |
| | | 950,400 | 864,000 | 950,400 |

## LIST OF REFERENCES


1. Rader, L. F., "Investigations of the Physical Properties of Asphaltic Mixtures at Low Temperatures", Proceedings of The American Society for Testing and Materials, Vol. 35, Part II, June 1936.

2. Tons, E. and Krokosky, E., "Tensile Properties of Dense Graded Bituminous Concrete", Proceedings of The American Association of Paving Technologists, Vol. 32, February 1963.

3. California Division of Highways, "Materials Manual of Testing and Control Producers", Vol. I, Sacramento, California, 1956.

4. Davis, W. C. and Jones, W. G., "Flexible Pavement Design by the Group Index Method", Highway Research Board Research Report 16-13, 1954.

5. Foster, C. R. and Ahllin, R. G., "Stresses and Deflections Induced by a Uniform Circular Load", Proceedings, Highway Research Board, 1954.

6. Palmer, L. A. and Barber, E. S., "Soil Displacement Under Circular Loaded Area", Proceedings, Highway Research Board, 1940.

7. Burmister, D. M., "The Theory of Stresses and Displacements in Layered Systems and Applications to the Design of Airport Runways", Proceedings, Highway Research Board, 1943.

8. Hveem, F. N. and Carmany, R. M., "The Factors Underlying a Rational Design of Pavements", Proceedings, Highway Research Board, 1948.

9. Highway Research Board, "The WASHO Road Test, Part II, Test Data Analysis Findings", Highway Research Board Special Report 22, 1955.

10. Highway Research Board, "The AASHO Road Test: Part 5-Pavement Research", Highway Research Board Special Report 61E, 1962.

11. Garniero, F. L. L. B. and Barcellus A., "Concrete Tensile Strength", Bulletin No. 13, International Association of Testing and Research Laboratories for Materials and Structures, Paris, March 1953.

12. Akazawa, T., "Tension Test Method for Concrete", Bulletin No. 16, International Association of Testing and Research Laboratories for Materials and Structures, Paris, November 1953.

13. Rusch, H. and Vigerlust, G., Discussion of "Tensile Splitting Test and High Strength Concrete Test Cylinders", A.C.I. Journal Proceedings, Vol. 54, No. 6, Part 2, December 1957, p. 1319.

14. McNeely, D. J. and Lash, S. D., "Tensile Strength of Concrete", A.C.I. Journal, Vol. 60, No. 6, Part I, June 1963, pp. 751-760.

15. Den Hartog, J. P., "Advanced Strength of Materials", McGraw-Hill Book Co., Inc., 1952.

16. Timoshenko, S. P., "Theory of Elasticity", McGraw-Hill Book Co., Inc., 1956.

17. Frocht, M. M., "Photoelasticity", Vol. 2, John Wiley and Sons, Inc., 1957.

18. Vesic, A. S. and Domaschuck, L., "Theoretical Analysis of Structural Behavior of Road Test Flexible Pavements", National Cooperative Highway Research Program Report 10, H.R.B., National Academy of Sciences, 1964.

19. Griffith, A. A., "The Phenomena of Rupture and Flow in Solids", Phil. Transactions of the Royal Society, London, A221, 1920.

20. Monismith, C. L., Secor, K. E., and Blackmer, E. W., "Asphalt Mixture Behavior in Repeated Flexure", Proceedings, Association of Asphalt Paving Technologists, Vol. 30, 1961, pp. 188-222.

21. Pell, P. S., "Fatigue Characteristics of Bitumen and Bituminous Mixes", Proceedings, International Conference on the Structural Design of Asphalt Pavements, 1962, pp. 310-323.

22. Jimenez, R. A., and Gallaway, B. M., "Behavior of Asphaltic Concrete Diaphragms to Repetitive Loadings", Proceedings, International Conference on the Structural Design of Asphalt Pavements, 1962, pp. 339-344.